Task 1 Report – Literature Review

Exploration of UHPC Applications for Montana Bridges

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1 Introduction

Past and current research into ultra-high performance concrete (UHPC) at Montana State University has been focused on: (1) creating a nonproprietary mix design (MT-UHPC), (2) the use of MT-UHPC for pile cap and shear-key connections on Montana bridges, and (3) implementation issues with MT-UHPC, such as batch sizes and temperature effects on curing. This report focuses on current/recent research on potential future bridge applications for MT-UHPC.

The specific tasks associated with this research are as follows:

Task 0 – Project Management

Task 1 – Literature Review

Task 2 – Material-Level Evaluation

Intermediate Technical Panel Meeting Task

Task 3 – Experimental Design of Structural Testing

Task 4 – Structural Testing

Task 5 – Analysis of Results and Reporting

This report documents the work completed as part of Task 1 – Literature Review. It should be noted that the document will continue to be updated as research progresses. This report outlines research that is pertinent to the current project and reflects on areas of research that may need further development. This report will be used to guide discussions during the Intermediate Technical Panel Meeting to ultimately decide the direction of the structural testing portion of this research (Tasks 3 and 4).

2 Background

UHPC became commercially available in the U.S. in 2000, and since then has been actively promoted by the Federal Highway Administration [1-6]. UHPC is generally understood to be a concrete with compressive strength of at least 20 ksi, post-cracking tensile strength of at least 0.72 ksi, and a discontinuous pore structure that improves durability by limiting permeability. These properties are achieved with: (1) low water-to-cement ratios, (2) aggregate gradations optimized for high particle packing density, (3) high quality aggregates and cements, (4) supplemental cementitious materials, (5) high particle dispersion during mixing, and (6) the incorporation of steel fiber reinforcement. Although the initial cost of UHPC far exceeds conventional concrete mixes, the use of UHPC has been shown to reduce life-cycle costs [7], as the increased durability of UHPC results in a longer service life and decreased maintenance costs. Further, the use of UHPC results in smaller/lighter structural elements.

Previous research conducted at MSU [8, 9] has included (1) the development of nonproprietary UHPC mixes that are significantly less expensive than commercially available mixes and are made with materials readily available in Montana, (2) an investigation into several items related to the field batching of these mixes, (3) an exploration into the potential variability in performance related to differences in constituent materials, (4) the investigation of rebar bond strength and the subsequent effect this has on development length, (5) an investigation on the effects of varying the mixing process, batch size, and mixing and curing temperatures, including the development of a maturity curve, (6) the use of MT-UHPC for precast pile cap joints and shear keys between precast deck elements on two bridges spanning Trail Creek on Highway 43, west of Wisdom, MT, and (7) and (in conjunction with the current study) the investigation of making a thixotropic version of the MT-UHPC mix. This previous research has been successful and has clearly demonstrated the feasibility of using MT-UHPC in Montana bridge projects.

The focus of the current project is to explore potential applications of MT-UHPC beyond its use in precast longitudinal joints and pile to pile-cap connections, and to conduct any additional testing to ensure its successful use in these applications. Some examples of research conducted to explore applications of UHPC in the U.S. include bridge pier seismic strengthening [10], 100% UHPC structural elements such as girders [11], composite slabs [12], and even precast applications [13]. Considering the aging infrastructure in the country, two of the more promising applications of UHPC are its use in thin-bonded overlays for bridge deck rehabilitation [14-17] and bridge member repairs, rehabilitation, and structural patching [18-22]. In the overlay application, UHPC not only provides enhanced structural performance, it also provides protection from chloride penetration and water ingression [15]. The main focuses of the current literature review are on the bridge deck overlay and bridge repair applications.

3 UHPC Bridge Deck Overlays

This section summarizes research on the use of UHPC as a bridge deck overlay. Specifically, this section discusses research projects conducted at Iowa State University, New Mexico State University, Missouri University of Science and Technology (Missouri S&T), and Montana State University.

3.1 Iowa State University & The Mud Creek Bridge

The first use of UHPC as an overlay in the United States was completed in 2016 on the Mud Creek Bridge on Buchanan County Road D48 near Brandon, Iowa [23]. This bridge is 102 ft long and 30 ft wide, is a continuous concrete slab bridge with two lanes, and has a 5% superelevation. Typically, UHPC is self-consolidating and therefore its use with superelevation is problematic. To accommodate this superelevation, a special thixotropic Ductal UHPC mix was produced by Lafarge Holcim by using thickening admixtures. Prior to the bridge application, Iowa State University performed a variety of tests to verify the performance and characteristics of this UHPC as an overlay material. These tests included prismatic slant shear tests and flexural tests, both on specimens with varying surface roughness to characterize the bond strengths. Figure 1 shows an example of a prismatic slant shear test, while Figure 2 shows a typical flexure test performed in this research. From the slant shear tests it was found that a minimum surface roughness of 0.125 in. gave the desired composite action, with the resulting failure occurring in the normal concrete (NC) layer. From the flexural tests, it was determined that a surface roughness of 0.25 in. resulted in the highest bond strengths, and the failure occurred in the NC.

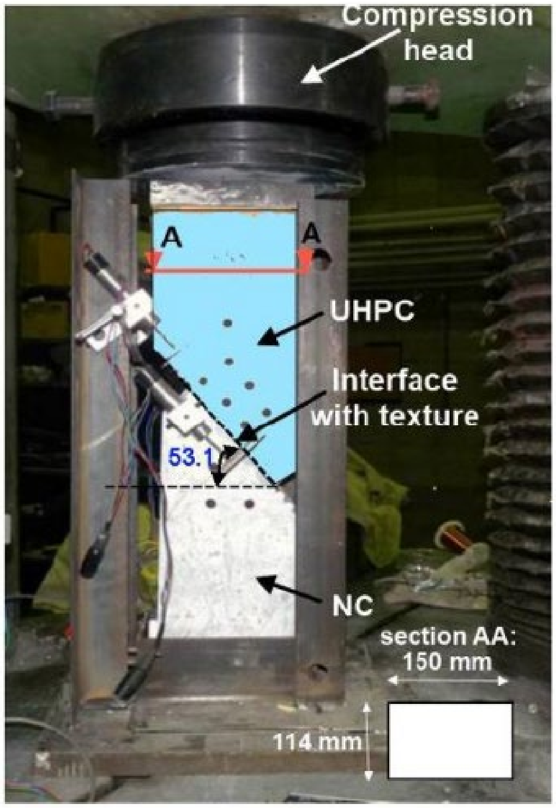


Figure 1: Example slant shear test setup [23]

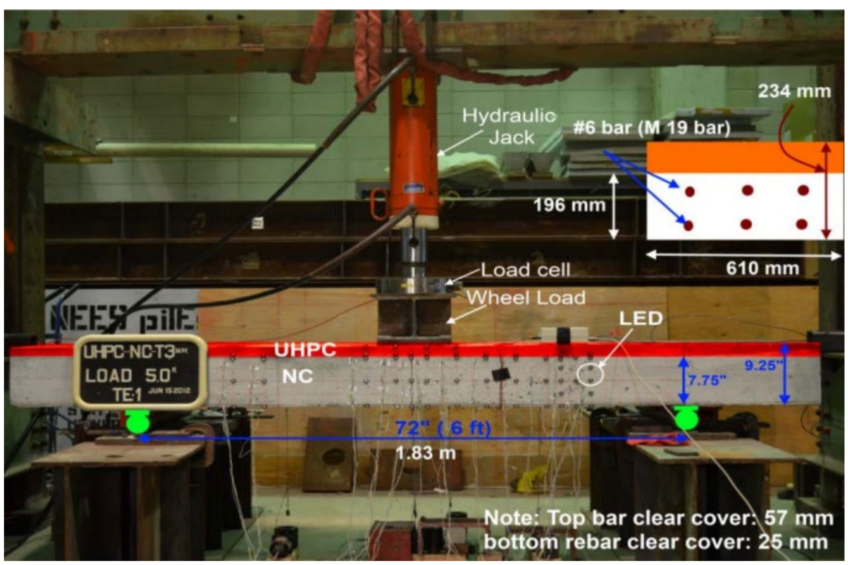


Figure 2: Example flexural test [23]

After the initial phases of this research, the thixotropic UHPC mix design was ready for field implementation on the Mud Creek Bridge. For this project, the top 0.25 in. of the deck surface was first removed, and the deck was then grooved along the bridge length with an amplitude of roughness ranging from one twelfth of an inch to one eighth. All batching and placing of the UHPC was performed on site by the contractor. A pair of high-shear pan mixers were used to mix the concrete. Each mixer had the capacity to mix 0.65 yd^3 (17.55 ft^3) of material. Loading and batching of the UHPC took approximately 20 minutes per batch. An overlay thickness of 1.5 in. was compacted and maintained by using a vibratory truss screed. All the mixing was done at one end of the bridge and transported using a mini concrete dumper. Grinding and grooving of the UHPC deck surface took place 4 days after placement (Figure 3), at which point the compressive strength had reached 12.3 ksi. Finally, the deck was evaluated using pull-off tests to quantify the bond strength between the UHPC and the substrate material.

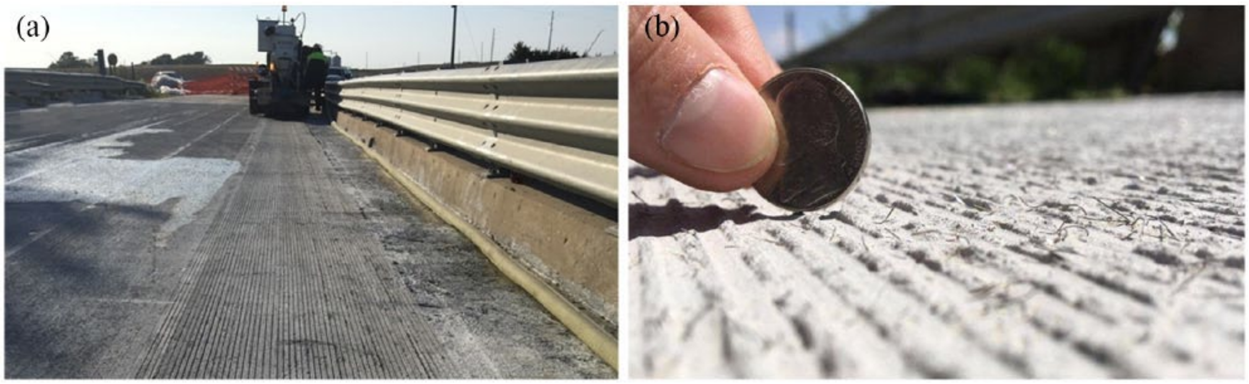
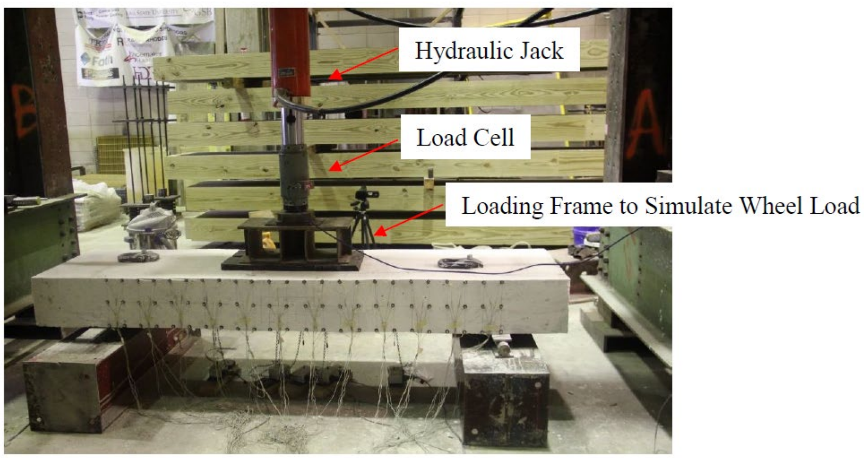


Figure 3: UHPC overlay on Mud Creek Bridge deck: (a) grooving of the surface; (b) closeup of finished surface [23]

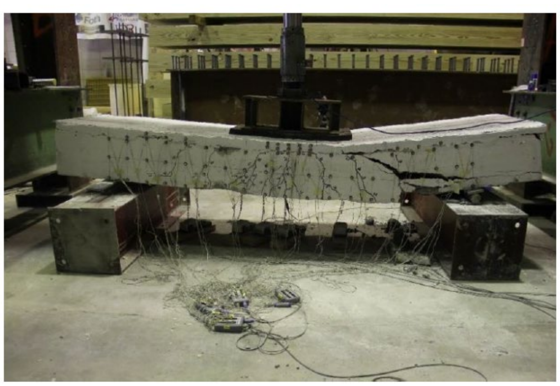
Additionally, the area over the pier locations was reinforced with a welded wire mesh to analyze the benefits of including this reinforcement in negative moment regions. After the construction was completed, a series of destructive and non-destructive tests were performed to ensure adequate bond strength between the UHPC and NC interface. Thermal imaging and the chain drag method were used first to identify eight potential delamination areas. Two out of the eight potential locations were then tested further using pull-off tests in accordance with the American Society of Testing and Materials standard (ASTM), ASTM C1583. Three additional areas that were determined to have good bond strengths were also tested for comparison. All tests resulted with failures in the NC layers, proving there was adequate bond strength between the NC and UHPC.

Two additional composite slabs with a wire mesh were cast using the UHPC mix from the bridge. This was done as a continuation of the flexure tests described earlier, but was focused on investigating the strength gain from using a wire mesh. The specimens were prepared similar to the negative moment sections of the bridge, though had to have the surface manually grinded instead of mechanically grooved like the bridge, due to the specimen size. The slabs being prepared are shown in Figure 4. The slabs were tested with a positive and negative moment and compared to an NC slab without an overlay. Results showed that both overlay specimens (positive and negative moment) showed increased strength and stiffness compared to the NC slab, though this was in part due to the additional 1.5-in. thickness from the overlay. The researchers concluded that the wire mesh did not add significant strength to the negative moment strength, due to the small amount of reinforcement added. A larger area of steel could lead to increased strengths, although could also affect the bond strength between the two layers. Figure 5 shows (a) the typical load setup used in this testing (similar to Figure 2), (b) the observed shear failure mechanism followed by partial UHPC debonding, and (c) the UHPC layer pried open after the test.

Figure 4: Casting UHPC on slab specimens [23]



a) Example concrete deck with UHPC overlay test setup



b) Partial overlay debonding following shear failure

c) UHPC layer pried open after the test

Figure 5: Observed failure of UHPC overlay flexural specimens [23]

3.2 New Mexico State University

New Mexico State University recently worked with the Transportation Consortium of South-Central States to create their own nonproprietary UHPC mix and evaluate its potential application as a bridge deck overlay [24]. The mix had an average compressive strength of 17.8 ksi. No comment was made about the flow of the proposed mix design. A series of tests were performed to determine the bond strength between the UHPC and NC layers. The tests included slant shear, split cylinder, split prism, and direct tension. Specimens were prepared by either grinding, adding horizontal grooves, adding cross-hatched grooves, or leaving a rough surface with a depth of 0.11 in. All the surface preparation techniques showed adequate strengths for the split cylinder and split prism tests, with strengths over ACI's recommendation of 150 psi for concrete repair [25]. However, none of these surface preparation techniques provided adequate strengths for the direct tension test. An additional chipped surface (not using a grinder) with a texture depth of 0.04 in. was tested after the completion of the original tests, and this method did provide adequate strengths. It was determined that the chipping method provided higher strengths because it did not plug the pores that help create the bond; however, this method is not recommended as it can damage the substrate layer more than expected. Overall, the bond assessment tests resulted in adequate bond strengths with textures (grinded or chipped) less than 0.08 in., which is lower than the minimum acceptable texture depth under field conditions of 0.25 in. from the American Concrete Institute (ACI) for conventional concrete repair [25], highlighting UHPC's high potential in overlay applications.

Both early-age and longer-term shrinkage was then investigated. A 6x6x24 in. beam was used to measure early-age shrinkage. It was found that 55% of shrinkage strain occurred while the UHPC was still plastic, and therefore this shrinkage strain would not cause a significant amount of horizontal shear stress between the UHPC overlay and the underlying NC deck. A 3x4x16 in. UHPC beam was used to measure long-term shrinkage. The beam was cured in a wet room for the first 7 days, then removed and cured in ambient conditions for the remainder of the 28 days. The shrinkage plateaued in the cure room around day 4. Outside the cure room, the shrinkage plateaued around day 20 with a max of about 450 µstrains.

To test the combined effects on shrinkage of composite UHPC-NC slabs, seven composite slabs were made with varying thicknesses of NC, exposure condition, steel reinforcement, and application of overlay. The NC slabs were cast first and at day 30 the surfaces were prepared with air hammers and chisels until the aggregates were exposed. Texture depth was measured following ASTM E965 and the average depths ranged from 0.06 in. to 0.15 in. Next, the UHPC overlays were cast and two photographs of the slab preparation are shown in Figure 6. Strains and external temperatures were measured over time. The results showed that the reinforcement of the NC layer had the greatest impact on reducing shrinkage caused by the UHPC overlay. However, NC layer thickness also played a role as thicker substrate slabs experienced more shrinkage than thinner slabs, with the same amount of steel; therefore, reinforcement ratio is key. Comparing the laboratory and outdoor exposure conditions, as expected, more uniform shrinkage was found for the laboratory specimens.

a) Placing the NC substrate

b) Finishing the UHPC overlay

Figure 6: Preparation of UHPC-NC slabs [24]

To evaluate the effects of strengthening a beam with UHPC, fatigue tests were first performed on a plain channel girder, which was then overlayed with UHPC as shown in Figure 7. The girder was first fatigue tested through 1,000 load-unload cycles to an approximate mid-span deflection of 0.4 in. and average load of 20.3 kips. After unloading the beam, a residual mid-span deflection 0.0516 in. remained. A 1.0 in. UHPC overlay was then added to the beam and the same loading cycles were repeated. After the addition of the overlay, the girder saw an increase in flexural strength and required an additional 5.45 kips to reach the same deflection. After unloading the beam, a residual mid-span deflection of 0.037 in. remained, though no visual cracking or debonding was observed. The beam was then loaded to failure. Cracking first occurred at a deflection of 0.53 in. and a load of 33.1 kips. The ultimate deflection and load were 5.99 in. and 90.7 kips, respectively. Even at ultimate loading, little to no cracking occurred in the UHPC layer with only isolated locations of delamination.

Figure 7: Channel Girder with UHPC overlay [24]

3.3 Missouri S&T

The Missouri Department of Transportation funded a research project at Missouri S&T on designing an optimized UHPC mix for bridge deck overlays [26]. Sixteen NC slabs were prepared to test different mix designs and thicknesses and compare to latex modified concrete (LMC). The slabs contained rebar mats near the top and bottom of the slab. The top surface was prepared using a chemical surface retarder and a stiff brush to expose the aggregate. The slabs were then left outside for 12 months prior to applying the overlay. Strain gauges, relative humidity sensors, and thermocouples were embedded between NC and UHPC layers (Figure 8). A life cycle cost analysis was also performed to compare UHPC and NC. The results showed that a 1.0 in. UHPC overlay was the most cost-effective based on deterministic and probabilistic results. Additionally, the cost of UHPC is likely to decrease as demand and production increases, making it more desirable as an overlay material in the future.

Figure 8: Example NC slabs with UHPC overlay and instrumentation [26]

3.4 Thixotropy for MT-UHPC

After investigating the use of UHPC as a bridge deck overlay material, there was an apparent need to adjust the current MT-UHPC mix design to exhibit thixotropic behavior. A thorough search was conducted on the topic of UHPC mix adjustments for thixotropy with little results, because all UHPC overlay implementation projects have used proprietary mixes. Montana State University [27] explored the following two potential methods for creating a thixotropic mix: 1) increase the amount of steel fibers and 2) use an additional admixture, and the overall results are summarized herein. The standard steel fiber amount used for MT-UHPC is 2% by volume and this was doubled to 4% for the thixotropic study. The following two different Master Matrix viscosity changing admixtures were tested: VMA 358 and UW 450. Overall, results showed that neither doubling the steel fibers nor adding the VMA 358 admixture yielded the desired effects on the overall viscosity of the MT-UHPC. However, the mix using UW 450 exhibited the desired increase in viscosity and was able to maintain its shape on a 6% slope. The increase in viscosity led to poor consolidation in the test cylinders, which in turn led to below average compressive strengths of only 13.58 ksi at 28 days. With respect to adjusting the MT-UHPC mix to exhibit thixotropic behavior, the initial results are promising. However, further research may be required to see how the thixotropic mix can be batched in a larger pan mixer, and if better consolidation in test cylinders will lead to higher strengths.

4 Bridge Repair Using UHPC (University of Connecticut)

This section summarizes research conducted at the University of Connecticut that explores using UHPC to repair existing bridge elements [28-30]. More specifically, they investigated a method for repairing the ends of steel girders using UHPC. This research consisted of three phases, which will be described in detail below.

4.1 Phase I – Rolled Girder Testing and FE Modeling

Phase I of their research focused on creating a UHPC repair method for the ends of deteriorated steel girders by testing three half-scale, rolled girders with varying amounts of deterioration [29]. The proposed repair method involved welding studs around the damaged portion of the web and flange, and then encasing the studs in UHPC. The three girders tested in this research consisted of an undamaged girder, a damaged girder, and a repaired girder. A 14-ft long W21x55 girder was selected as the test girder size because it was approximately half the scale of a W36x160 bridge girder commonly used in Connecticut and it has the same web slenderness ratio. To simulate corrosion in the two deteriorated test girders (damaged and repaired), the end of the lower tee of the girders were removed using a plasma cutter. A portion of the web and flange were then milled off using a computer numeric controlled (CNC) milling machine. Full penetration groove welds were then used to attach the lower tee section back to the girder for the damaged and repaired specimens. The tee for the repaired section was not attached until after the studs were welded on. Nelson Stud Welding H4L Headed Concrete Anchors, 0.375 in. diameter by 1.25 in. long, were used as scaled down equivalents of the standard 0.75 in. size used in typical composite steel and concrete decks. The studs were staggered vertically and horizontally on opposite sides of the web to avoid bearing stress concentrations. The studs were then encased in a 25 in. long, 13 in. high, 1.75 in. deep, panel of UHPC. The stud pattern and UHPC panel are shown in Figure 9. The UHPC panel was formed using R-10 foam board and plexiglass. The girder and formwork were coated in mineral oil prior to casting to represent the lack of bond strength with girder paint. Mineral oil was not applied to the studs. A JS1212 Ductal mix provided by LaFarge Holcim was used for the UHPC.

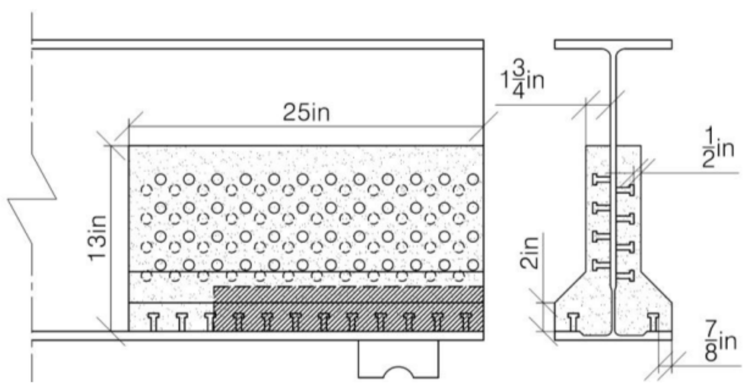
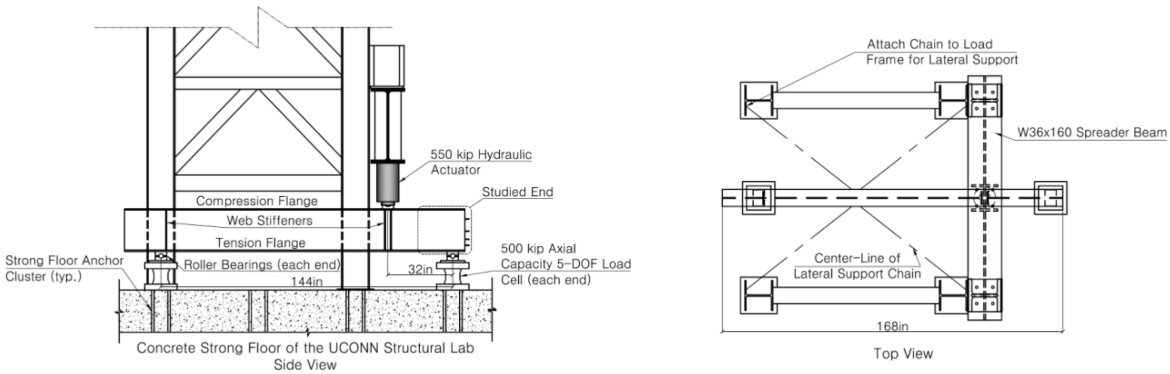


Figure 9: UHPC Panel for the Large-Scale Repair [29]

All three girders were tested using the same loading setup shown below in Figure 10. Web stiffeners (0.5 in.) were added at the loading point and the end not being tested, but not to the testing end. Additionally, the top of the beam was laterally restrained near midspan using clevises and chains. The web of the girder near the testing end was also treated with limestone and water to produce a thin white layer that would flake off during straining of the web and highlight any damage accrued during testing. The undamaged girder failed at a load of 180 kips through web buckling at the end of the girder, over the entire height of the bearing. The damaged girder failed at a load of 43.4 kips due to instability in the web at the top of the damaged section. The repaired girder reached a maximum load of 230 kips, where it experienced extensive flexural yielding but did not fail. All three girders after testing are shown below in Figure 11. The repaired girder was able to hold over five times the capacity of the damaged girder, and over 28% of the undamaged girder. All three girders had similar stiffnesses.



a) Side View b) Top View

Figure 10: Large-Scale Experiment Setup [29]

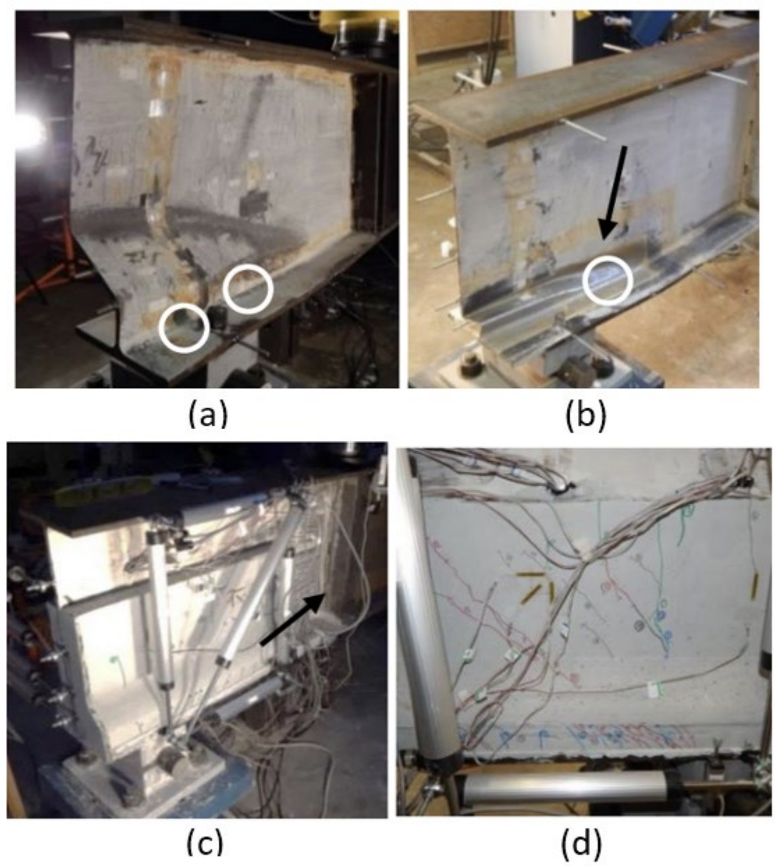


Figure 11: Conditions of each specimen after testing (a) Undamaged, (b) Damaged, (c) Repaired, (d) UHPC Cracking [29]

A finite element (FE) model was then created to predict the capacities of steel girders repaired using a variety of UHPC strengthening details. The model was first compared to the results from the half-scale tests to validate its accuracy, then it was used to evaluate several UHPC repair methods not tested in this research to evaluate their efficacy. The model was shown to accurately predict the failure modes and shapes; however, it potentially predicted inaccurate stress concentrations making the model conservative. Eight new repair methods were then evaluated across three types of girders with the FE model. The eight repair methods were created for full-height, half-height, and L-shaped repair; and the three girders evaluated were rolled girders without stiffener, rolled girder with stiffener, and a plate girder. All repair methods were shown to increase the capacity compared to the undamaged girder.

4.2 Phase II – Stud Testing and Model Improvement

To improve the accuracy of the FE model, Phase II of the research evaluated the effectiveness of various repair details (e.g. stud layout, concrete cover, underlying steel condition) [28]. To test the strength of the studs on older degraded steel, smaller scale push-off tests were performed on rolled steel girders that were salvaged from an old bridge. For the push-off tests, studs were welded to the web section of the salvaged girders and then encased in UHPC as shown in Figure 12. Eight specimens were made by varying the stud size, layout, spacing, concrete cover, and concrete type (UHPC and NC). The same JS1212 Ductal UHPC mix provided by LaFarge Holcim from Phase I was used for all UHPC specimens. The specimens were compressed with one end bearing on the girder and the other end on the concrete to analyze the stud failure. The test setup is shown in Figure 13.

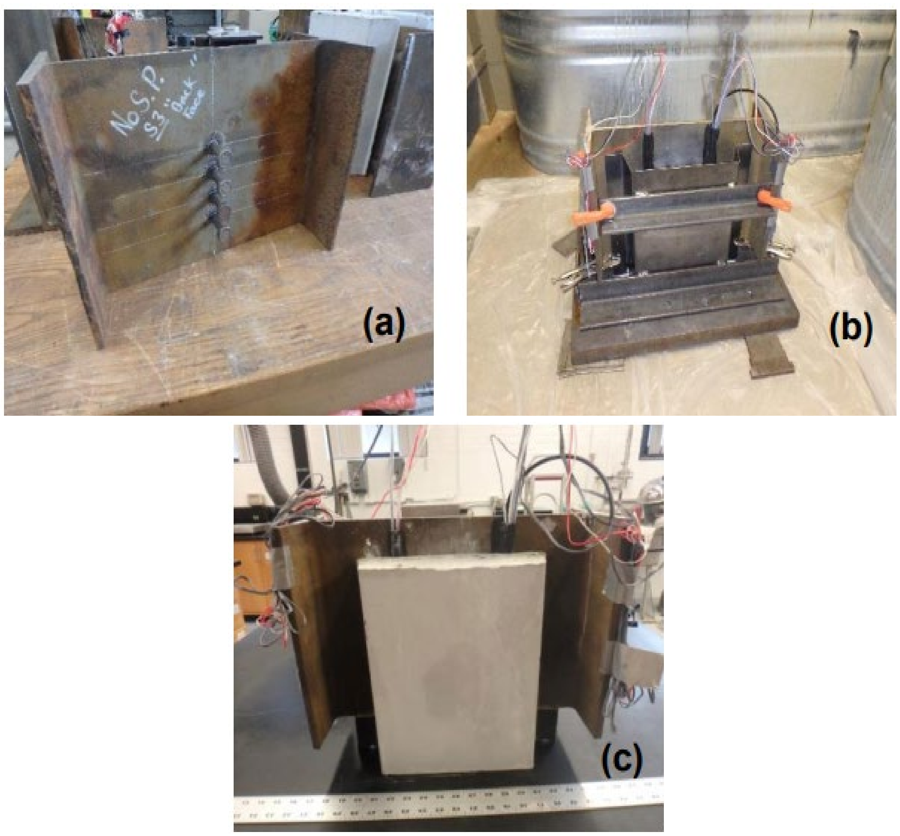


Figure 12: (a) Beam prior to casting; (b) beam with formwork used for casing concrete, and (c) completed push-off sample [28]

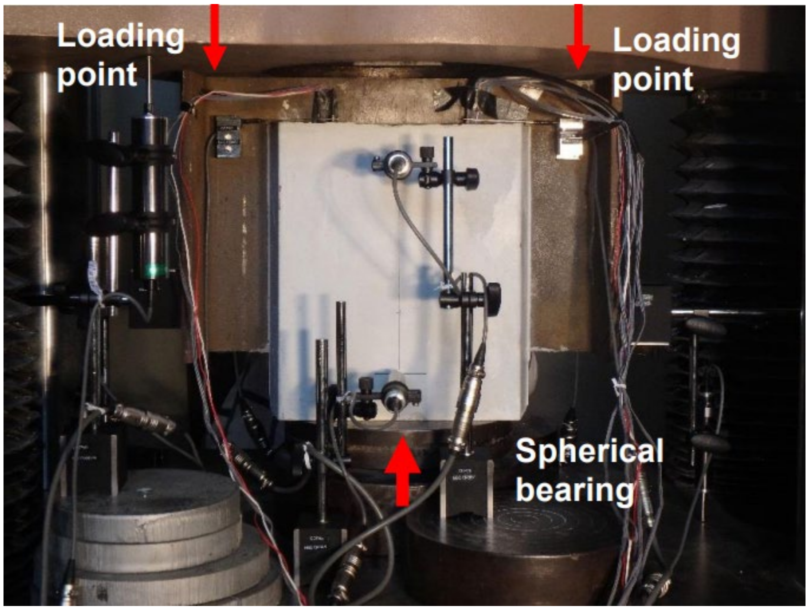


Figure 13: Typical experimental setup for push-off test [28]

The NC specimen was deemed unsuitable for the envisioned repair method as it split in tension when larger loads were applied. Conversely, all UHPC specimens failed through shear failure of the stud shank, with little to no cracking in the UHPC, and exceeded the theoretical capacity outlined by AASHTO [31]. Results showed that the 0.5-in. diameter studs performed the best, with the 0.675-in. diameter studs having a 25% reduction in capacity when compared to the 0.5-in. diameter studs. The changes in layout and spacing had little effect on the overall strength of the capacity of the specimens. Three-dimensional scans were also used to more accurately model the corroded girders in the FE model. A 3D scanner was used to create a point cloud of the corroded girder specimens, the point cloud was imported into an FE model, and ultimately, the methodology was shown to accurately model the corroded section and stress concentrations.

4.3 Phase III – Full-Scale Repair and Testing

Phase III of the research was focused on applying the developed repair method on four full-scale plate girders [30]. The design of the full-scale plate girder was chosen to represent an average bridge in Connecticut. A permanent girder was designed with a splice near one end to allow for a section that could be connected to replaceable test panels for each tested girder specimen as seen below in Figure 14. Overall, the testing setup was similar to that of the girders tested in Phase I, though upscaled to account for the larger plate girders. Local suppliers provided and fabricated the steel girders using grade A36 steel plates. Corrosion was simulated using sand blasting to give a non-uniform section loss near the ends of the girder. Other methods such as electrochemical corrosion and CNC milling were also considered but did not produce as desirable of results compared to the sand blasting.

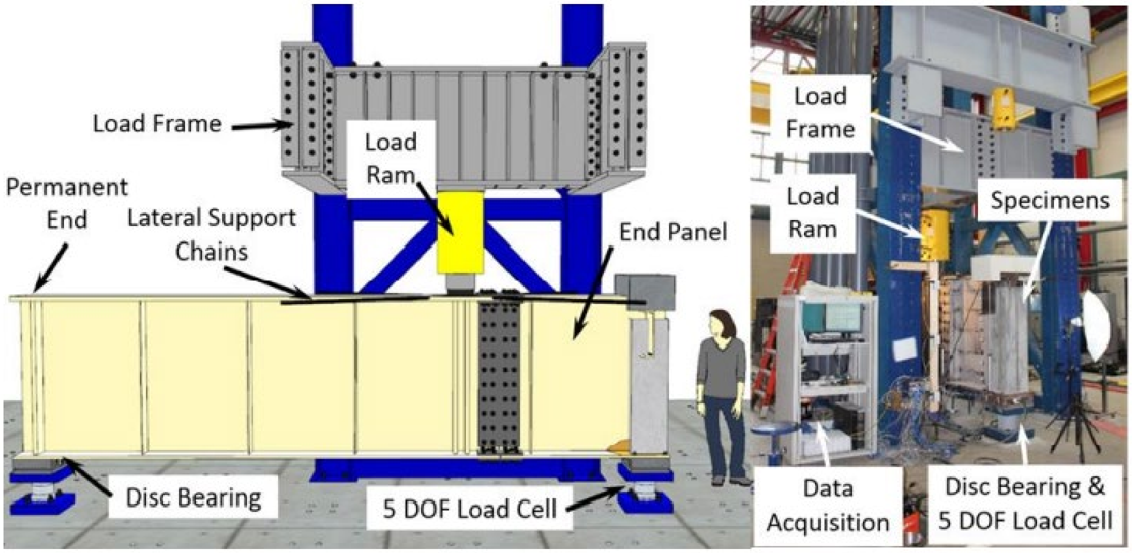


Figure 14: Testing setup [30]

Two full-height repairs (Full Height 1 and Full Height 2), one half-height repair (Half Height), and one baseline damaged girder were tested. The difference between the two full-height repairs were the stud layouts and UHPC used. Full Height 1 used Ductal JS1000, a slower setting and higher strength mix, whereas Full Height 2 used Ductal JS1212, a faster setting and lower strength mix that was also vibrated to simulate vehicle traffic on a bridge. Half Height used Ductal JS1212 (same as Full Height 2) and only covered the lower half of the web. The number of studs to be welded to the web were determined by dividing the estimated nominal shear capacity of the plate girder by the design shear capacity of a stud. This resulted in 28 studs being used in varying layouts for the three repaired specimens. The Full Height and Half Height repair drawings are shown in Figure 15.

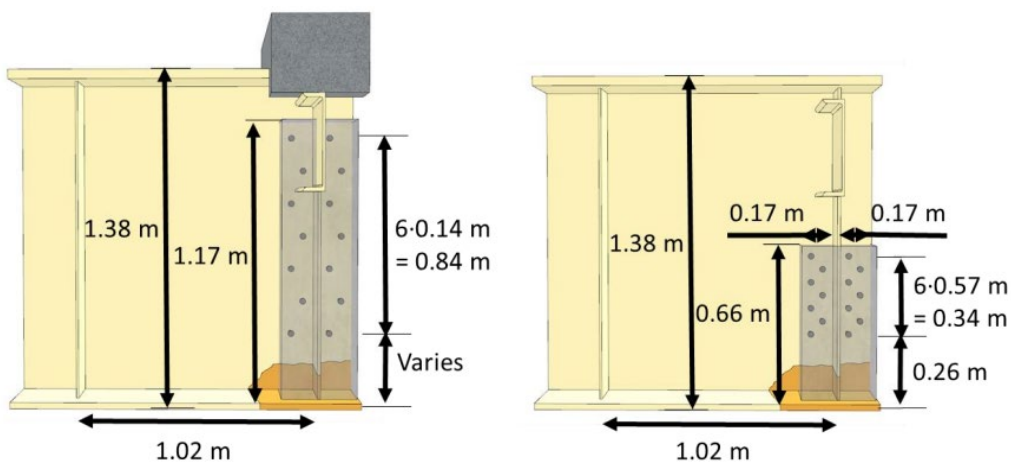


Figure 15: Full Height and Half Height Repairs [30]

The baseline corroded plate girder reached a flexural capacity of 95 kips and failed due to localized buckling of the web and bearing stiffener in the corroded region. The Full Height 1 repaired girder reached an initial peak capacity of 527 kips at a deflection of 0.504 in.; afterwards however, the girder was then able to sustain 450 kips up to a deflection of 0.994 in. where it failed due to web buckling of the end panel. Full Height 2 and Half Height performed very similarly to Full Height 1 with post-peak sustained load capacities at similar deflections. Full Height 2 reached initial peak and post-peak sustained load capacities of 497 kips and 450 kips, respectively, whereas Half Height reached respective capacities of 472 kips and 400 kips despite only covering half the cross section. The beams tested in this research were then modeled using the FE method developed and refined in the previous research phases. The predicted responses from this method were very similar to the measured responses from the tests. Based on the results, the half-height repair with 0.5-in. headed shear studs was recommended for use in repairs moving forward. Despite having a lower capacity, the method allows for easier construction and requires less UHPC. Additionally, the study concluded that although AASHTO's predicted values for shear stud capacity are conservative, they should still be used in order to account for uncertainties in weld quality.

5 Summary of Literature Review Findings

UHPC has been successfully used in several projects and across several applications, as described in this report. For the bridge deck overlay application, the literature review findings reveal that other state DOTs are almost exclusively using proprietary mixes during implementation projects. This is most likely a result of the same implementation issues being researched in the current work at MSU, including batch sizing for large volumes and the need for thixotropic behavior in this specific application. For example, the UHPC overlay implementation project conducted by Iowa State University used a special thixotropic Ductal UHPC mix produced by Lafarge Holcim. Additionally, adequate bond strengths have been found between UHPC and NC substrates across a variety of surface preparation techniques, with NC surface preparations below minimum depth requirements.

A repair method using shear studs and UHPC for the ends of steel girders was investigated in depth by the University of Connecticut. This method was shown to work well. Initial testing that compared an undamaged girder, a damaged girder, and a repaired girder, showed that while all three girders had similar stiffnesses, the repaired girder was able to hold over five times the capacity of the damaged girder and over 28% of the undamaged girder. Based on the comparisons made during the full-scale testing, a half-height repair with 0.5-in. headed shear studs was recommended, because although it has a lower capacity than a full-height repair, strengths are adequate, the method allows for easier construction, and less UHPC is required.

Overall, while UHPC bridge deck overlays are promising, additional research is still required to evaluate fine-tuning and further investigating the thixotropic version of MT-UHPC that would be required if a proprietary mix is not used. Using UHPC to repair and strengthen existing bridge elements is of interest and although very promising results have been shown for steel girder end repairs, other types and methods of repair could be investigated. MSU researchers look forward to discussing the direction of the structural testing portion of the current research project with the MDT Technical Panel.

6 References

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